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EFFECTS OF MOISTURE ON LIME-STABILIZED LATERITIC SOIL

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ABSTRACT

This study investigated effect of moisture content on the lime stabilized lateritic soil in Oyo-West Local Government, South-Western Area, Nigeria to determine the suitability and lime stabilization requirements of selected lateritic soil samples as pavement construction material.

The soil samples material were collected from the borrow pits within the area and subjected to laboratory tests such as California Bearing Ratio Test (CBR), Unconfined Compressive Strength (UCS), compaction test, Atterberg's Limit Test and sieve analysis in accordance with the British Standard BS1377 (1990) while the stabilization test were performed in accordance with BS1924(1990).

The grain-size analysis showed the percentage sieve No. 200 of 41.4%, this indicates low clay content sample. The liquid limit and Plastic Index values range from 9.5 and 70% and 3 and 32% respectively. Also, the Maximum Dry Density (MDD) ranges from 1.78 and 2.10 g/cm^3 and Optimum Moisture Content (OMC) 9 and 18%. The soaked and unsoaked CBR values ranges from 30 and 50% and 52 and 70%. The Unconfined Compressive Strength (UCS) increased from 146.75 and 605.75kN/m^3 for the lime-stabilized soil.

In conclusion samples with lime additive cured for 6 days with water absorption rate reduced from 51.94 to 50.41% under the same condition. The lime treatment of lateritic soils is however a remedial measure to improve the strength of soil material for road construction works n water-logged areas.

KEYWORDS: Moisture Content, Lateritic Soil and Stabilization

INTRODUCTION

One of the major reasons for structural failure particularly in pavement design is the non-availability of generalized relevant data of the particular soils involved in the area of construction. Various cases of under design of pavement strength have been recorded lately due to assumptions on sub-grade properties involved and which resulted into early failure. To give a cost-effective pavement should be constructed over good sub-grade materials. Therefore this will remove early failure such as; portholes, raveling, shoving, rutting and so on.

However, pavements that will perform well can adequately designed for and constructed over very poor sub-grade materials that have little or no compaction through improvement on the thickness of the overlaying materials (sub-base, base course) and or stabilization process (Osula, 1991).

Soil Stabilization

For many years, various forms of materials including products with varying degrees of purity have been utilized successfully as soil stabilizing agents. However, hydrated high calcium lime $Ca(OH)_2$, Monohydrated Dolomitic Lime Ca

$(OH)_2$, MgO, calcitic quicklime CaO. MgO are most frequently used (Pery, 2005). Although lime hydrates dominate the US market, quicklime use has increase over the past 20 years and currently accounts for 25 percents of the total stabilization lime on an annual basis. Many significant engineering properties of soils are beneficially modified by the soil treatment. Although lime is primarily utilized to treat fine-grained soils, it can also be used to modify the characteristics of the fine traction of more granular soils. Soil treatment can expedite construction, modify sub-grade soils, and improve strength and durability of fine-grained soils. Treated soils have been used as modified sub-grade, sub-base materials, and base materials in pavement construction. The location of the treated layer in the pavement system is dictated by strength, durability, and other design criteria. Railroad sub-grades have also been successfully stabilized with different materials (Dallas et al, 2008).

Mechanisms of Lime-Stabilization

According to Mustapha, (2005) and Lovetoknow (2006), Soil-Lime reactions are complex; however, understanding of the chemistry involved and results of field experience are sufficient to provide design guidelines for successful lime treatment of a range of soils. The sustained (and relatively slow) pozzolanic reaction between lime and soil silica and soil alumina (released in the high-PH environment) is key to effective and durable stabilization in lime-soil mixtures. Mixture design procedures that secure this reaction must be adopted. In addition to stabilizing materials, lime plays an increasing role in the reclamation of road bases. Lime has been used effectively to upgrade or reclaim not only clay soils, but also clay-contaminated aggregate bases and even calcareous bases that have little or no appreciable day. Work in the United States, South Africa, and France has established the benefits of lime stabilization of calcareous bases into rigid systems that could be susceptible to cracking and shrinkage.(Ogunsanwo, 1988 and 1989).

Mixture Design, Pavement Design, and Performance Considerations of Lime Stabilization

Design of Lime-stabilized mixtures is usually bases on laboratory analysis of desired engineering properties. Several approaches to mix design currently exist. In addition to engineering design criteria, users must consider whether the laboratory procedures used adequately simulate field conditions and long-term performance. Aspects of these procedures are likely to be superceded as the American Association of State Highway and Transportation officials (AASHTO) shifts to a mechanistic-empirical approach. Laboratory testing procedures include determining optimum lime requirements and moisture content, preparing samples, and curing the samples under simulated field conditions. (Little, 1995, Smith, 1991).

Curing is important for chemically stabilized soils and aggregates-particularly lime-stabilized soils-because lime-soils reactions are time and temperature dependent and continue for long periods of time (even years). Pozzolanic reactions are slower than cement-hydration reactions and can result in construction and performance benefits, such as extended mixing times in heavy clays (more intimate mixing) and autogenous healing of moderately damaged layers, even after years of service. On the other hand, longer reactions may mean that traffic delays are associated with using the pavement. In addition, protocols for lime-soil mixture design must address the impact of moisture on performance. Lime stabilization construction is relatively straight forward. In-place mixing (to the appropriate depth) is usually employed to add the proper amount of lime to a soil, mixed to an appropriate depth. Pulverization and mixing are used to combine the lime and soil thoroughly. For heavy clays, preliminary mixing may be followed by 24 to 28 hours (or more) of moist curing prior to final mixing. This ability to “mellow” the soil for extended periods and then remix is unique to lime. During this process, a more intimate mixing of the lime and the heavy clay occurs, resulting in more complete stabilization. For maximum development of strength and durability, proper curing is also important. Other methods of lime stabilization include in-plant mixing and pressure injection. (National Lime Association, 2004).

MATERIALS AND METHODS

Lateritic soil samples were collected at ten locations (10) within the study area and were subjected to the following laboratory tests.

California Bearing Ratio (CBR) Test

This test would be carried out based on the optimum moisture content and maximum dry density of the soils deduced from the dynamic compaction process.

Unconfined Compressive Strength Test (UCS)

The shear strength of the soil samples were determined by Unconfined Compressive Strength Test (UCS). It depends on the MDD and OMC of both the lateritic soil and lime-stabilized lateritic soil.

Atterberg's Limit Test

This test consists of two basic tests i.e. Liquid Limit (LL) and Plastic Limit (PL). Where LL is described as the water content at which soil possesses an arbitrary fixed small amount of shear strength and it is the water content that represents the boundary between the liquid and plastic state of soil. And PL Is the moisture content at which a thread of the soil sample (abt. 3mm diameter) begins to rupture or crumble when it is being tried to be moulded. Plasticity Index = Liquid Limit – Plastic Limit (PL = LL – PL)

Compaction Test (BS Proctor)

Compaction test used to determine the optimum-moisture content (OMC) and maximum dry-density (MDD) of the soil sample.

Sieve Analysis

This test was determined by carrying sieve analysis to know the grain sizes and classified soil type.

RESULTS

California Bearing Ratio (CBR)

Table1 shows the summary of the CBR test results for both soaked and unsoaked (CBR) conditions. Values for all the samples increased with an increase in the lime percentage but dropped after optimum lime content was reached. This further corroborates the result obtained in the compaction test. The results showed that the CBR values for the unsoaked condition of the stabilized soils reduces after soaking as a result of the absorption of water which weakened the soil. The difference between the soaked and unsoaked CBR value can be associated with the PI of the stabilized soils at the respective lime contents which determines their swell potential, the higher the PI, the higher the difference between the unsoaked and soaked value. The following unsoaked and soaked CBR value increased from 63.82 and 48.14 at 6%, 53.70 and 39.50 at 12%; 58.00 and 44.89 at 15% then 51.94 and 50.41 at 18% lime content respectively.

It can be observed that in their unstabilized state, the difference between the unsoaked and soaked CBR values of the soil is quite large compared to the values in their lime stabilized conditions, this is because in their natural states water could still percolate into the interstitial spaces of the soil thereby weakening them. However, this is reduced in their optimum-lime stabilized state as it has effectively bonded the soil particles to form a closely packed mass that results the ingress of water.

The result obtained shows that the strength of the subgrade in term of load bearing capacity for the soaked condition is between 30.83 and 50.41% for 48hrs. The soaked CBR ranges from 6, 4, 12, 7, 8, and 28% are suitable and normal for the subgrade material in accordance with the specification of the Federal Ministry of Works and Housing which specify 3-10% maximum for subgrade materials while the value 50.41 at 18% lime exhibit the highest strength value of soaked CBR. The unsoaked CBR between 51.94 and 70% and % lime 0-70% merit the standard. According to the Federal Ministry of Works and Housing. However, it can be noted that the CBR value of 51.94 and 50.41 at 18% lime is extremely low compared to other value at their percentages. It's unstable behaviour in the liquid limit states.

Table 1: Result of California Bearing Ratio (CBR) Result Relatively to Percentage (%) Lime Additive (0-28)

% LIME – ADDITIVE	UNSOAKED CONDITION (CBR %)	SOAKED CONDITION (CBR %)
0	61.14	30.28
3	68.12	31.91
6	63.82	48.14
9	57.33	35.96
12	53.70	39.50
15	58.00	44.89
18	51.94	50.41
21	67.25	37.63
24	52.45	30.83
28	70.30	42.73

Unconfined Compressive Strenght (UCS)

The Unconfined Compressive Strength determines the strength of the soil sample. The testing strength of the soil improves by increasing the percentage of lime additive, that is, an increase in lime ratio, the strength of lateritic soil tends to improve. Therefore, from the Table 2, the addition of lime to be the lateritic soils improves the strength by increasing the UCS from 146.75N/m^2 to 605.75N/m^2 for BS energy standard proctor curbed with polyethylene for 6 days. It was observed that an addition of lime create an enhancement in the strength. Therefore lime additive/stabilizer enhances the strength of a lateritic soil.

Table 2: Results of Unconfined Compressive Strength of Lime-Stabilized Lateritic Soil

Lime Additive (%)	Axial Strain Dial Reading (mm)	Axial Load Dial Reading (N)									
		A	B	C	D	E	F	G	H	I	J
0	20	6	7	8	9	10	11	12	14	16	19
3	40	20	25	28	22	29	24	24	27	36	29
6	60	39	39	39	35	42	38	32	40	49	46

Table 2: Contd.,

9	80	43	46	46	41	58	52	49	59	54	66
12	100	56	58	58	54	77	83	58	99	54	98
15	120	70	76	76	73	84	142	72	128	68	136
18	140	81	84	84	85	95	230	88	354	85	239
21	160	86	115	87	92	120	308	99	229	138	306
24	180	85	130	96	92	69	288	130	320	197	320
28	200	84	95	94	99	115	274	169	275	270	298
31	220	79	80	89	89	60	266	210	284	95	276
34	240	68	68	66	74	105	250	150	280	120	255
UCS		**146.75**	**221.67**	**166.28**	**170.20**	**204.96**	**526.07**	**362.95**	**605.75**	**459.55**	**554.27**

Table 2: Cont'd

%Lime Additive	Bulk Density	Dry Density	UCS (N/m^2)	Shear Strength
0	1.73	1.60	146.75	73.36
3	1.54	1.41	221.67	110.835
6	1.54	1.41	166.28	83.14
9	1.83	1.65	170.20	85.10
12	1.97	1.67	204.96	102.48
15	1.85	1.64	526.07	263.04
18	1.55	1.34	362.95	362.95
21	1.77	1.50	605.75	302.88
24	1.83	1.63	459.55	459.55
28	1.95	1.73	554.27	277.14

Atterberge's Limit Test

The Table 3 illustrates the summary of the liquid limits (LL) result of the soils at the required percentage lime from 0, 3, 6, 9, 12, 15, 18, 21, 24 and 28% and their values ranges from 38, 7, 18, 17, 5, 10, 31, 20, 38 and 7 respectively and also Plastic Index (PI) of 0%(11), 3%(10.5), 9%(7), 12%(3.5), 15%(4), 18%(14), 21%(15.5), 24%(32) and 28%(3) respectively. The addition of lime to the soil sample produces a corresponding increase in the Liquid Limit (LL) and Plastic Limit (PL) of the soil, this causing a decrease in its PI especially in 0, 9, 18, 21, and 24% of lime order with their corresponding values. And another category reacts differently by the decrease in its PL. The addition of lime to sample caused a decrease in their swell potentials. The reduction in the swell potential is as a result of the cation exchange which occurs when Ca^{2+} ions from the lime replace weaker cation in the soil, thereby causing a better sealing of the voids by the agglomeration of the particles and this has a positive effect on the soils strength properties.

Table 3: Summary of Atterberg Limits (for Lime 0-28 %)

%Lime Additive	Moisture Content (%)			No of Blows			Average PL	Average LL	Average PI
0	45.10	51.16	56.52	32	21	17	38.00	49.0	11.0
3	8.77	15.38	12.06	35	24	15	7.00	12.0	5.0
6	22.22	29.55	37.70	33	24	16	18.00	28.5	10.5
9	21.84	26.13	28.16	30	23	16	17.00	24.0	7.0
12	7.69	12.50	19.56	28	17	12	6.00	9.5	3.5
15	9.56	8.42	13.40	27	11	15	10.00	13.5	4.0
18	39.13	44.44	50.00	38	22	11	31.00	45.0	14
21	30.68	36.36	39.18	32	24	18	20.00	35.5	15.5
24	66.66	69.23	76.0	34	23	13	38.00	70.0	32.0
28	9.30	12.20	14.29	27	18	12	7.00	10.0	3.0

Compaction Test at BS Standard

The summary of the compaction test result as presented in Table 4, dry density (MDD) vary between 1.78 and 2.1g/cm^3 and optimum moisture content (OMC) ranges between 9 and 18%. From the observation the (OMC) of the soil sample increases with increase in lime content. This can be linked with the additional water needed to enable the Pozzolanic soil lime reactions necessary for the stabilization process.

MDD increased as the percentage of lime increased to an optimum value after which it falls. The maximum value represents the optimum percentage of lime required for stabilization.

The increase in MDD was as a result of increasing lime particles that were ready to perform the exchange of cations with the soil particles, thus filling-up voids spaces and densely packing the soil particles together. However, the drop in density resulted from the excess water and lime remaining after the increasing quantity has been used up for a stabilization process. For OMC increases from 9.0% at 6% to 16.0% at 7% with a corresponding increase in MDD from 1.92g/cm^3 to 1.14g/cm^3 at 9% lime content, and at the same time OMC increases from 10.0% and 1.97g/cm^3 at 15% to 12.6% and 2.05g/cm^3 at 24% and the other one which increases from 18.0% and 1.78g/cm^3 to 14.0% and 1.92g/cm^3 at 0 and 3% respectively. These can thus be stated that these are the optimum percentages of lime for the soil sample.

Table 4: Compaction Test Result Based on Percentages of Lime Additive

Lime Additive (%)	0%	3%	6%	9%	12%	15%	18%	21%	24%	28%
Dry Density g/cm^3	1.61	1.80	1.69	1.79	1.68	1.70	1.76	1.65	1.71	1.84
	1.81	1.98	2.03	2.01	1.92	1.97	1.95	1.78	1.95	2.09
	1.94	1.92	2.05	1.85	1.28	1.82	1.74	1.53	1.72	1.96
	1.49	1.67	1.76	1.57	1.66	1.59	1.67	1.43	1.67	1.54
Moisture Content (%)	5.45	5.76	8.09	6.84	5.80	8.87	7.94	13.95	6.90	6.21
	10.34	12.10	5.07	12.57	14.10	10.92	10.32	18.02	11.87	9.31
	15.93	16.00	8.86	14.09	17.29	20.8	19.84	24.23	12.03	12.21
	24.08	21.40	18.43	19.10	20.58	27.7	21.35	28.93	17.10	26.53

Sieve Analysis

According to the specification speculated by the Federal Ministry of Works and Housing in Nigeria (FMWH, 1997) for the grain size distribution of particles. It was stated that the percentage passing BS sieve No. 200 should not be greater than 35% and the sample 23.4% merit the standard. from the Table 5, the result obtained shows that the soil sample fall within the range of A4 – A7 according to ASSHTO classification system, that is, they are fair to poor soils.

Table 5: Result of Sieve Analysis Relatively to the Order of Percentage Lime-Additive

Lime Additive (%)	Sieve Size	Sieve Sample Mass(G)	Empty Sieve Mass (G)	Sieve Dia (mm)	Mass Retained (G)	Percentage Retained (%)	Cumulative Percentage Retained (%)	Percentag e Passing (%)
0	¾ inch	480.65	480.65	20.000	0	0.0	0.0	100.0
3	5/6 inch	566.08	464.08	8.000	102	20.4	20.4	79.6
6	No. 5	501.75	437.75	4.000	64	12.8	33.2	66.8
9	No. 10	456.20	412.20	2.000	44	8.8	42.0	58.0
12	No. 18	441.10	386.10	1.000	55	11.0	53.0	47.0
15	No. 40	374.92	336.92	0.425	38	7.6	60.6	39.4

Table 5: Contd.,

18	No. 60	365.60	315.60	0.250	50	10.0	70.6	29.4
21	No. 120	320.50	295.50	0.125	23	4.6	75.2	24.8
24	No. 200	295.05	288.05	0.075	7	1.4	76.6	23.4
28	No. 200			<0.075	117	23.4	100.0	0.0

Table 6: Physical Properties of Soil Sample

(%) Lime Additive	% Passive No. 200BS	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	MDD (g/cm^3)	OMC (%)	Unsoaked Condition (CBR%)	Soaked Condition (CBR%)	Bulk Density (eW)	Dry Density (ed)	UCS (N/m^2)	Shear Strength	AASHTO Classification	Remarks
0	23.4	49	38	11	1.78	18	60.14	30.28	1.73	1.6	146.75	73.36	A4-A7	Fair poor
3		12	7	5	1.92	12	68.12	31.91	1.54	1.41	221.67	110.835		
6		28.5	18	10.5	1.92	14	63.82	48.14	1.54	1.41	166.28	83.14		
9		24	17	7	1.94	16	51.33	35.96	1.83	1.65	170.2	85.1		
12		9.5	6	3.5	1.95	10.3	53.7	39.5	1.97	1.67	204.96	102.48		
15		13.5	10	4	1.97	10	58	44.89	1.85	1.64	526.07	263.04		
18		45	31	14	1.98	12	51.94	50.41	1.55	1.34	362.95	362.95		
21		35.5	20	15.5	2.01	12.6	67.25	37.63	1.77	1.5	605.75	302.88		
24		70	38	32	2.05	9	52.46	30.83	1.83	1.63	459.55	459.55		
28		10	7	3	2.1	9	70.3	42.13	1.95	1.73	554.27	277.14		

CONCLUSIONS

The laboratory test that was carried out on the lime-stabilized latritic soil are determined for the suitable of road construction works as regards the Federal Ministry of Works and Housing (FMWH, 1997) standards.

The study revealed that the presence of moisture content reduces the strength of lateritic soil. On the other hand the addition of lime to the lateritic soils generally reduces the soils plasticity and water absorption capability which led to the improvement on the strength characteristics of lateritic soil oven in the presence of moisture.

However, the study on the effect of moisture content on the lime-stabilized lateritic soil using the BS compactive efforts was achieved and showed that the strength of lateritic soil increases with addition of lime or stabilizer.

Conclusively, the addition of lime modifies and enhances the properties of lateritic soils.

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